

# CHAPTER 9. DETENTION FOR FLOOD CONTROL

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## **1.0 OVERVIEW**

### **1.1 Detention Requirement**

In accordance with the Springfield City Code, Chapter 36 Land Development Code, Section 410 and Chapter 96 Storm Water, Section 96-14 Detention, the post-development peak runoff rate for all land development must be no greater than the pre-development peak runoff rate for the design storms listed in this chapter. In most cases, meeting this standard requires the design and construction of detention in accordance with the criteria contained in this chapter. Additional design considerations may be necessary where water quality criteria must be met (See Chapter 10, Water Quality). Detention basins must be safe, be maintainable, enhance the environment, and provide aesthetic value to the community. The incorporation of multipurpose uses including open space, recreation, water quality enhancement, and habitat is strongly encouraged.

### **1.2 Payment in Lieu of Constructing Detention**

In accordance with the Springfield City Code, Chapter 96 Storm Water, Section 96-18 Detention, when the developer's engineer can show that constructing a detention basin will provide no downstream benefits and no existing downstream drainage problems will be impacted, a payment in lieu of constructing detention may be acceptable. An Application for Payment in Lieu of Constructing Storm Water Detention **MUST** be completed, signed, and sealed by a qualified design professional to be considered (See Appendix A).

### **1.3 Design Spreadsheets**

The SF-Detention Spreadsheet included with this manual provides multiple worksheets to assist in design calculations. Worksheets are included to complete the following calculations:

- Detention volume using the Modified FAA (Federal Aviation Administration) Method
- Detention volume using a hydrograph
- Stage-Storage for triangular ponds, rectangular basins, elliptical ponds, and ponds with irregular geometry
- Collection capacities for horizontal and vertical orifices (inlet controls)
- Flow capacity of a riser (inlet control)
- Capacities of circular and box culverts (inlet vs. outlet control with tailwater effects)
- Stage-discharge through a spillway

## 2.0 TYPES OF DETENTION

There are two basic approaches to designing storage facilities. Runoff storage facilities planned on an individual-site basis are referred to as onsite or private facilities. Facilities that are planned to serve multiple lots, a subdivision, or larger area are referred to as common or regional facilities.

Requirements for detention basins may vary depending on the development situation, including:

1. **Single Lot Commercial:** Generally, these are developments on lots that are not part of a subdivision. Basins shall be designed for full development of the lot based on zoning unless land use restrictions dictate less land is available for development. Construction of detention may be phased when only part of the lot is proposed to develop.
2. **Residential or Commercial Subdivision:** These are developments that involve the subdivision of property. One or more basins may be required depending on natural drainage patterns.
3. **Multiple Properties:** Multiple properties or developments may be served by a regional basin that is not within the boundary of the development.

Generally, the type of detention is determined by the required design objectives and the appearance and function desired by the developer. Detention basins may fall into one of the following design types.

1. **Dry:** Designed for several different frequency rainfalls for flood control only and drains over a relatively short period of time. The outlet is typically made up of orifices and/or weirs.
2. **Extended:** Designed for pollutant removal and possibly flood control and drains over an extended period of time, typically one to three days. The outlet is typically made up of a filtered control as well as orifices and/or weirs.
3. **Wet:** Contains a permanent pool of water and is designed for pollutant removal, flood control, and often aesthetics. May be designed to drain down to the permanent pool level over a short or long period of time.

Unplanned storage may also be present in features such as sinkholes and the upstream side of railroad and highway embankments. When planning a development along a major waterway, upstream unplanned storage should be accounted for when calculating existing flow rates but generally should not be accounted for when calculating ultimate future peak flow rates. Exceptions include storage that is dedicated for perpetuity and has adequate assurances for long-term maintenance.

In developments where an offsite area drains across the property, the developer must consider whether to construct the basin in-line and direct offsite runoff through the basin or construct it off-line and convey the offsite runoff so that it bypasses the basin. In-line and off-line storage are defined as:

1. **In-Line Storage:** A facility located in-line with the drainageway that captures and routes the entire flood volume. A disadvantage with in-line storage is that it must be large enough to store and convey the total flood volume of the entire tributary catchment, including offsite runoff, if it exists. A U.S. Army Corps of Engineers (USACE) Section 404 permit for dredge and fill activities within the waters of the United States and a Section 401 Water Quality Certification from the Missouri Department of Natural Resources (MDNR) are typically required for in-line storage.
2. **Off-Line Storage:** A facility located off-line from the drainageway that receives runoff from a smaller drainage area or from a particular site. These facilities are often smaller and potentially store water less frequently than in-line facilities.

For all types of basins, the designer should consider safety, aesthetics, and multipurpose uses during both wet and dry conditions. The use of other specialists such as architects, biologists, and planners is encouraged to achieve these objectives.

### 3.0 HYDROLOGIC AND HYDRAULIC DESIGN

#### 3.1 Design Methods

Three design methods acceptable for use in detention design are summarized in Table DET-1, depending on the detention volume and impervious area added. When determining which method is acceptable, the calculated volume takes precedence over the impervious area added.

**Table DET-1**  
**Acceptable Detention Design Methods**

Detention Design Method	Acceptable Volume (cubic feet [ft <sup>3</sup> ])	Approx. Acceptable Impervious Area Added
Tabular Method (Section 3.1.1)	< 5,000 ft <sup>3</sup>	½ acre
Simplified (Modified FAA) Method (Section 3.1.2)	<20,000 ft <sup>3</sup>	2 acres
Hydrograph Method (Section 3.1.3)	Any size	Any amount

##### 3.1.1 Tabular Method for Detention Volumes Less than 5,000 ft<sup>3</sup>

Table DET-2 may be used to determine detention volumes less than 5,000 ft<sup>3</sup>. To determine the required volume, cross-reference the area of the site in acres in the left column with the percentage of impervious area on the top row. This method may be used to determine the required volume for a payment in lieu of detention or the required volume for construction of a basin.

**Table DET-2**  
**Tabular Method for Determining Detention Volumes Less than 5,000 ft<sup>3</sup>**

		Percent Impervious Added								
		20%	30%	40%	50%	60%	70%	80%	90%	100%
		Required Detention Volume (ft <sup>3</sup> )								
Area of Site (acres)	0.10	250	390	500	640	765	900	1,020	1,150	1,280
	0.20	500	765	1,020	1,275	1,530	1,800	2,040	2,300	2,550
	0.30	765	1,150	1,530	1,920	2,300	2,680	3,060	3,450	3,830
	0.40	1,020	1,530	2,040	2,550	3,060	3,570	4,080	4,600	
	0.50	1,280	1,910	2,550	3,200	3,830	4,460			
	0.60	1,530	2,300	3,060	3,830	4,600				
	0.70	1,800	2,680	3,570	4,460					
	0.80	2,040	3,060	4,080						
	0.90	2,300	3,450	4,590						
	1.00	2,550	2,830							
	1.25	3,190	4,780							
	1.50	3,830								
	1.75	4,470								

Detention basins designs must include a low-flow orifice designed to discharge at the 1-year peak flow rate and an overflow spillway designed to convey the 100-year post-development flow rate above the required volume. Peak flow rates must be calculated using the Rational Method (Chapter 5, Calculation of Runoff). The low-flow orifice and overflow spillway must be designed based on methods given in Section 3.3.2 of this chapter. The low-flow orifice must be a minimum of 6 inches in diameter. Overflow spillways using turf reinforcement mat or sod are preferred over concrete. A freeboard of 6 inches must be provided.

### 3.1.2 Simplified Method for Detention Volumes Less than 20,000 ft<sup>3</sup>

An acceptable simplified method of detention design for volumes less than 20,000 ft<sup>3</sup> is the FAA Method (FAA 1966) as modified by Guo (1999a). It is acceptable to use hydrograph routing procedures for the sizing of facilities smaller than 20,000 ft<sup>3</sup>, provided the inflow hydrograph runoff rates are consistent with the peak flow rates obtained using the methods described in Chapter 5, Calculation of Runoff, and the maximum allowable release rates given in this chapter. For more detail on detention design using the Modified FAA Method, see Section 3.2.

### **3.1.3 Hydrograph Method for Large Detention Basins**

For detention volumes greater than 20,000 ft<sup>3</sup>, a hydrograph method must be used (i.e., Kinematic Wave Method or Soil Conservation Service [SCS] Unit Hydrograph Method with the appropriate Huff rainfall distribution, as described in Chapter 5, Calculation of Runoff). If detention facilities or other significant storage basins are located upstream, hydrograph routing methods should be used to account for their effects. If offsite tributary areas contribute runoff to an onsite detention facility, the total tributary area must be included in the sizing of the onsite storage volumes, with the assumption that offsite land uses are fully developed, in order to account for the total runoff volume in the watershed.

### **3.1.4 Determining Detention Storage for Extended Detention Basins**

See Chapter 10, Water Quality, for design procedures for extended detention to meet water quality requirements. When extended detention is incorporated into the basin, the water quality capture volume (WQCV) may also be used for flood control volume (i.e., the basin may be assumed to be empty at the start of the level pool routing calculation in Section 3.3).

## **3.2 Sizing of Onsite Detention Facilities**

### **3.2.1 Peak Flow Attenuation Requirements for Onsite Facilities**

These criteria for peak flow attenuation apply for onsite facilities unless other rates are recommended in a City-approved master plan. Three conditions must be examined for determination of attenuation requirements for an onsite facility:

1. Condition 1—Pre-project conditions
2. Condition 2—Post-project conditions
3. Condition 3—Fully urbanized conditions for the entire tributary watershed with no upstream detention

Onsite detention facilities must be designed so that peak flow rates for post-project conditions (Condition 2) are limited to pre-project levels (Condition 1) for the design events specified in this chapter. Condition 3 is applied for overflow spillway design. Condition 3 requires analysis for the 100-year event (to assure safe conveyance). If downstream safety considerations warrant, it may be necessary to size a spillway for greater than a 100-year event.

### **3.2.2 Simplified Method for Volume Estimation**

The simplified method for volume estimation uses the rational formula based Modified FAA procedure to calculate a reasonable estimate of storage volume requirements for onsite detention facilities. This method can be applied to multiple design events for a site to determine storage requirements for various

return intervals. This method may also be used for initial sizing of detention storage volumes whenever a detailed hydrograph routing design method is used.

The inputs required for the Modified FAA volume calculation procedure include:

$A$  = Area of the catchment tributary to the storage facility (acres)

$C$  = Runoff coefficient (unitless)

$Q_{po}$  = Allowable maximum peak outflow rate from the detention facility based on pre-project conditions or City-approved master plan release rates (cfs)

$t_c$  = Time of concentration for the tributary catchment (see Chapter 5, Calculation of Runoff) (minutes)

$i$  = Rainfall intensity corresponding to  $t_c$  for relevant return frequency storms (as determined from the intensity-duration-frequency table in Chapter 5, Calculation of Runoff) (in/hr)

As shown by example in Section 5.1 (Table DET-4), the calculations are best set up in a tabular (spreadsheet) form. The Springfield FAA worksheet in the SF-Detention spreadsheet can be used for this purpose. Each time increment (typically 5 minutes) is entered in rows, and the following variables are entered or calculated in each column:

1. Storm Duration Time,  $t$  (minutes), up to 180 minutes. For longer durations, a hydrograph-based method is required.
2. Rainfall Intensity,  $i$  (inches per hour), based on the intensity-duration-frequency table in Chapter 5, Calculation of Runoff.
3. Inflow volume,  $V_i$  (ft<sup>3</sup>), calculated as the cumulative volume at the given storm duration using the equation:

$$V_i = CiA (60 t) \quad \text{(Equation DET-1)}$$

4. Outflow adjustment factor,  $m$  (Guo 1999a):

$$m = \frac{1}{2} \left( 1 + \frac{t_c}{t} \right) \quad 0.5 \leq m \leq 1 \quad \text{and} \quad t \geq t_c \quad \text{(Equation DET-2)}$$

5. The calculated average outflow rate,  $Q_{av}$  (cfs), over the duration  $t$ :

$$Q_{av} = mQ_{po} \quad \text{(Equation DET-3)}$$

6. The calculated outflow volume,  $V_o$  ( $\text{ft}^3$ ), during the given duration and the adjustment factor at that duration calculated using the equation:

$$V_o = Q_{av}(60t) \quad (\text{Equation DET-4})$$

7. The required storage volume,  $V_s$  ( $\text{ft}^3$ ), calculated using the equation:

$$V_s = V_i - V_o \quad (\text{Equation DET-5})$$

The value of  $V_s$  increases with time, reaches a maximum value, and then starts to decrease. The maximum value of  $V_s$  is the required storage volume for the detention facility.

### 3.2.3 Outlet Works Design

To maintain peak flow rates at pre-development levels, a multi-frequency outlet design approach is required. The designer must demonstrate that the 1-, 10-, and 100-year post-development peak flow rates are limited to the corresponding pre-development flow rates. Because the FAA Method calculates required volume only, Section 3.3.2 must be used to design the outlet. The designer must show that the volume design and outlet design are compatible with the calculated volume for each design event to ensure peak discharges do not exceed pre-development rates for each design event (e.g., for the water surface elevation corresponding to the volume calculated for the 10-year event, the outlet should be designed to discharge no greater than the 10-year pre-development peak flow rate). If the facility is also providing water quality treatment, then the detention volume and outlet design must also incorporate the WQCV (See Chapter 10, Water Quality).

## 3.3 Reservoir Routing of Storm Hydrographs for Sizing of Storage Volumes

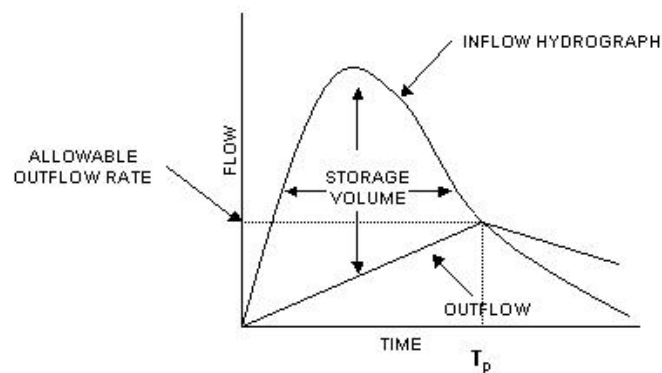
Detention facilities larger than 20,000  $\text{ft}^3$  (typically 5 acres of residential development or 2.5 acres of commercial development) must use the hydrograph sizing procedure described in this section.

### 3.3.1 Detention Volume Estimation

The required detention volume for a proposed development may be estimated using a variety of methods, including the Rational Formula Based Modified FAA Method or the Hydrograph Volumetric Method. The Rational Formula Based Modified FAA Method may be used to find an initial storage volume for any size watershed. This technique for initial sizing yields best results when the tributary watershed area is less than 300 acres, but can be applied to larger watersheds, although the design volumes may need to be adjusted significantly.

It is possible to use the inflow hydrograph, along with desired maximum release rates, to make an initial estimate of the required storage volume using the Hydrograph Volumetric Method. This technique

assumes that the required detention volume is equal to the difference in volume between the inflow hydrograph and the simplified outflow hydrograph. This is represented by the area between those two hydrographs from the beginning of a runoff event until the time that the allowable release occurs on the recession limb of the inflow hydrograph (Guo 1999b) (see Figure DET-1). Generally, the inflow hydrograph is obtained from a hydrograph method using the Huff distribution presented in Chapter 5, Calculation of Runoff. The outflow hydrograph can be approximated using a straight line between zero at the start of the runoff to a point where the allowable discharge is on the descending limb of the inflow hydrograph,  $T_p$ .



**Figure DET-1**  
**Hydrograph Volumetric Method of Detention Volume Sizing**

The volume can be calculated by setting up tabular calculations, as shown by example in Table DET-5 (Section 5.2). The Hydrograph worksheet in the SF-Detention spreadsheet can also be used to complete these calculations. Descriptions of the variables in the table columns include:

1. The time,  $T$  (in minutes), from 0 to  $T_p$  in uniform increments. Time increments ( $\Delta t$ ) of 5 minutes are typically used.  $T_p$  is the time (in minutes) where the descending limb of the inflow hydrograph is equal to the allowable release rate.
2. The inflow rate,  $Q_i$  (cfs), to the detention basin corresponding to the time  $T$ . The inflow rate can be obtained using the SCS Unit Hydrograph Method with the Huff distribution presented in Chapter 5, Calculation of Runoff.
3. The outflow rate,  $Q_o$  (cfs), calculated as:

$$Q_o = \frac{T}{T_p} Q_{po} \quad (\text{Equation DET-6})$$

In which:

$Q_{po}$  = the peak outflow rate. The allowable peak outflow rate is determined from City criteria or a City-approved master plan.

4. The incremental storage volume,  $V_s$ :

$$V_s = (Q_i - Q_o) \cdot \Delta t \cdot 60 \text{ seconds} \quad (\text{Equation DET-7})$$

5. The total cumulative storage volume calculated as the sum of the values in column 4.

$$V_{s\text{total}} = \sum V_s \text{ incremental} \quad (\text{Equation DET-8})$$

### 3.3.2 Outlet Works Design

The hydraulic capacity of the various components of the outlet works (i.e., orifices, weirs, pipes) can be determined using standard hydraulic equations. The discharge pipe of the outlet works functions as a culvert. See Chapter 7, Bridges and Culverts, for guidance regarding the calculation of the hydraulic capacity of outlet pipes. To create a rating curve for the entire outlet, a composite total outlet rating curve can be developed based on the rating curves developed for each of the components of the outlet and then selecting the most restrictive element that controls the release at a given stage. Several worksheets for various types of outlets are provided in the SF-Detention spreadsheet to aid in calculations for outlet works designs.

#### 3.3.2.1 Orifices

Single or multiple orifices may be used in a detention facility and are commonly used as a low-flow control. The hydraulics of each can be superimposed to develop the outlet rating curve. The basic orifice equation is:

$$Q = C_o A_o (2gH_o)^{0.5} \quad (\text{Equation DET-9})$$

In which:

$Q$  = the orifice flow rate (cfs)

$C_o$  = discharge coefficient (0.60 for concrete openings)

$A_o$  = area of orifice ( $\text{ft}^2$ )

$H_o$  = effective head on the orifice (ft)

$g$  = gravitational acceleration (32.2 ft/s<sup>2</sup>)

If the orifice discharges as a free outfall, the effective head is measured from the centroid of the orifice to the upstream water surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces.

### 3.3.2.2 Weirs

Several different types of weirs may be used, as described below.

**Rectangular Sharp-Crested Weirs:** A sharp-crested weir is defined as a weir with a wall thickness of 6 inches or less. The basic equation for a rectangular sharp-crested weir is:

$$Q = CL_{eff}H^{3/2} \quad \text{(Equation DET-10)}$$

In which:

$Q$  = discharge (cfs)

$L_{eff}$  = effective horizontal weir length (ft) (as calculated in Equation DET-11 to account for contractions)

$$L_{eff} = L_{total} - 0.1NH \quad \text{(Equation DET-11)}$$

In which:

$N$  = number of contracted sides

$L_{total}$  = the total weir height (ft)

$H$  = head above weir crest excluding velocity head (ft)

$C$  = weir coefficient (as calculated in Equation DET-12 or DET-13)

The weir coefficient is a function of the head above the weir crest,  $H$ , and the height of the weir crest above the pond or channel bottom,  $H_c$ . For ratios of  $H/H_c$  up to approximately 10, the following equation should be applied to determine  $C$  (Debo and Reese 2003):

$$C = 3.237 + 0.428 \frac{H}{H_c} + 0.0175H \quad \text{(Equation DET-12)}$$

For ratios of  $H/H_c$  greater than 15, the weir coefficient is found using:

$$C = 5.68(1 + \frac{H_c}{H})^{1.5} \quad (\text{Equation DET-13})$$

For ratios of  $H/H_c$  between 10 and 15, the designer should interpolate between Equations DET-12 and DET-13.

**Broad-Crested Weirs:** The equation for a broad-crested weir is:

$$Q = CLH^{3/2} \quad (\text{Equation DET-14})$$

In which:

$Q$  = discharge (cfs)

$C$  = broad-crested weir coefficient from Table DET-3

$L$  = broad-crested weir length (ft). (For weirs with tapered sides, it is acceptable to use a length equal to the average of the upper and lower weir lengths.)

$H$  = head above weir crest (ft)

**Table DET-3**  
**Broad-crested Weir Coefficients**

Depth (ft)	C for 6-inch Thick Wwall	C for 8-inch Thick Wall	C for 12-inch Thick Wall
0.20	2.80	2.77	2.69
0.25	2.83	2.79	2.70
0.30	2.86	2.80	2.71
0.40	2.92	2.84	2.72
0.50	3.00	2.90	2.74
0.60	3.08	2.95	2.75
0.70	3.19	3.03	2.80
0.75	3.25	3.08	2.83
0.80	3.30	3.12	2.85
0.90	3.31	3.16	2.92
1.00	3.32	3.20	2.98
1.25	3.32	3.25	3.11
1.50	3.32	3.29	3.24
1.75	3.32	3.31	3.27
2.00	3.32	3.32	3.30
2.50	3.32	3.32	3.31
> 2.5	3.32	3.32	3.32

**Slot and V-Notch Weirs:** Capacity of broad-crested slot and V-notch weirs shall be determined by the following equation developed by Joe Wilson at the University of Missouri-Rolla:

$$Q = 0.86H + (3.65W + 5.82z)H^{1/2} \quad (\text{Equation DET-15})$$

In which:

$Q$  = discharge (cfs)

$H$  = upstream head (ponded depth above the slot invert) (ft) (maximum of 6 ft)

$W$  = slot invert width perpendicular to flow (ft) ( $0.333 < W < 2.0$ )

$z$  = slope of slot sides expressed in terms of H:1V ( $0 < z < 0.6$ )

### 3.3.3 Reservoir Routing

The City recommends the use of the Modified Puls method for reservoir routing for the design of detention facilities. This reservoir routing method determines an outflow hydrograph for a detention facility based on a given inflow hydrograph and the storage-outflow characteristics of a facility. This method is typically carried out using computer programs such as HEC-HMS, TR-20 or proprietary software packages. Model input is typically a storage-outflow relationship for the detention facility. This section provides background on the Modified Puls method and shows an example calculation to assist the engineer in understanding the reservoir routing method. The description is adapted from *Fundamentals of Hydraulic Engineering Systems* (Hwang and Houghtalen 1996).

#### 3.3.3.1 Modified Puls Method

The mathematical basis of Modified Puls routing is the continuity equation (conservation of mass with constant density). Simply stated, the change in storage is equal to inflow minus outflow. In differential format, the equation can be expressed as:

$$\frac{dS}{dt} = I - O \quad (\text{Equation DET-16})$$

In which:

$dS/dt$  = rate of change of storage with respect to time

$I$  = instantaneous inflow

$O$  = instantaneous outflow

If average rates of inflow and outflow are used, an acceptable solution can be obtained over a discrete time step ( $\Delta t$ ) using:

$$\frac{\Delta S}{\Delta t} = \bar{I} - \bar{O} \quad (\text{Equation DET-17})$$

Where  $\Delta S$  is the storage change over the time step. By assuming linearity of flow across the time step, the storage equation may be expressed as:

$$\Delta S = \left[ \frac{(I_i + I_j)}{2} - \frac{(O_i + O_j)}{2} \right] \cdot \Delta t \quad (\text{Equation DET-18})$$

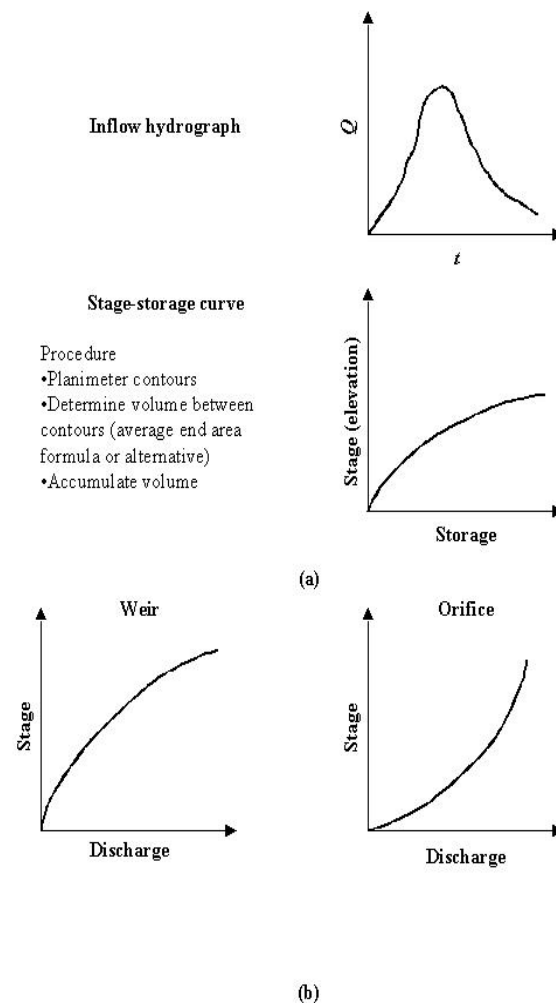
Where the subscripts  $i$  and  $j$  designate inflow and outflow at the beginning and end of the time step, respectively.

The storage relationship in Equation DET-18 has two unknowns. Because the inflow hydrograph must be defined prior to performing the routing calculations (using the SCS Unit Hydrograph Method with the Huff rainfall distribution), inflow values ( $I_i$  and  $I_j$ ) are known. Likewise, the time increment ( $\Delta t$ ) is chosen, and outflow at the beginning of the time step ( $O_i$ ) was solved in the previous time step calculations (or specified as an initial value). That leaves the storage increment ( $\Delta S$ ) and the outflow at the end of the time step ( $O_j$ ) as unknowns. Because both storage and outflow (for uncontrolled outlet devices) are related to the depth of water in the detention facility, they are related to one another. This relationship is employed to compute the solution.

The data requirements to perform Modified Puls reservoir routing include:

1. An inflow hydrograph (determined using the SCS Unit Hydrograph Method or the Kinematic Wave Method using the Huff rainfall distribution).
2. An elevation versus storage relationship for the detention facility. The Stage-Storage worksheets in the SF-Detention Spreadsheet can be used to develop this relationship.
3. An elevation versus outflow relationship. The Stage-Discharge worksheets in the SF-Detention Spreadsheet can be used to develop this relationship.

Figure DET-2 displays these data requirements graphically. The procedure for obtaining the stage (elevation) versus storage curve is described in the figure. Also, the two basic types of outlet devices (weirs and orifices) are noted with typical stage-discharge relationships.



**Figure DET-2**  
**Data Requirements for Storage Routing (after Hwang and Houghtalen 1996)**

The Modified Puls routing method reformulates Equation DET-18, as shown by Equation DET-19:

$$(I_i + I_j) + \left[ \frac{2S_i}{\Delta t} - O_i \right] = \left[ \frac{2S_j}{\Delta t} + O_j \right] \quad (\text{Equation DET-19})$$

Where  $(S_j - S_i)$  equals the change in storage ( $\Delta S$ ). The advantage of this expression is that all of the known values are on the left side and all of the unknowns are grouped on the right. The solution procedure for Modified Puls routing is as follows:

1. Determine the appropriate inflow hydrograph for the detention facility (see Chapter 5, Calculation of Runoff).
2. Select a routing interval ( $\Delta t$ ). Linearity of inflows and outflows over the time step is assumed.

3. Determine stage-storage and stage-outflow relationships for the detention facility and outlet device(s) selected. (See SF-Detention Spreadsheet for assistance in these calculations.)
4. Establish the storage-outflow relationship by setting up a table with the following headings (note that headings b through e correspond with variables in Equation DET-19:
  - a. Elevation
  - b. Outflow (O)
  - c. Storage (S)
  - d.  $2S/\Delta t$
  - e.  $2S/\Delta t + O$
5. Plot the  $(2S/\Delta t + O)$  versus O relationship.
6. Perform routing using a table with the following headings:
  - a. Time
  - b. Inflow at time step i (I<sub>i</sub>)
  - c. Inflow at time step j (I<sub>j</sub>)
  - d.  $2S/\Delta t - O$
  - e.  $2S/\Delta t + O$
  - f. Outflow

See Section 5.3 for an example calculation.

## **4.0 FINAL DESIGN CONSIDERATIONS**

### **4.1 Potential for Multiple Uses**

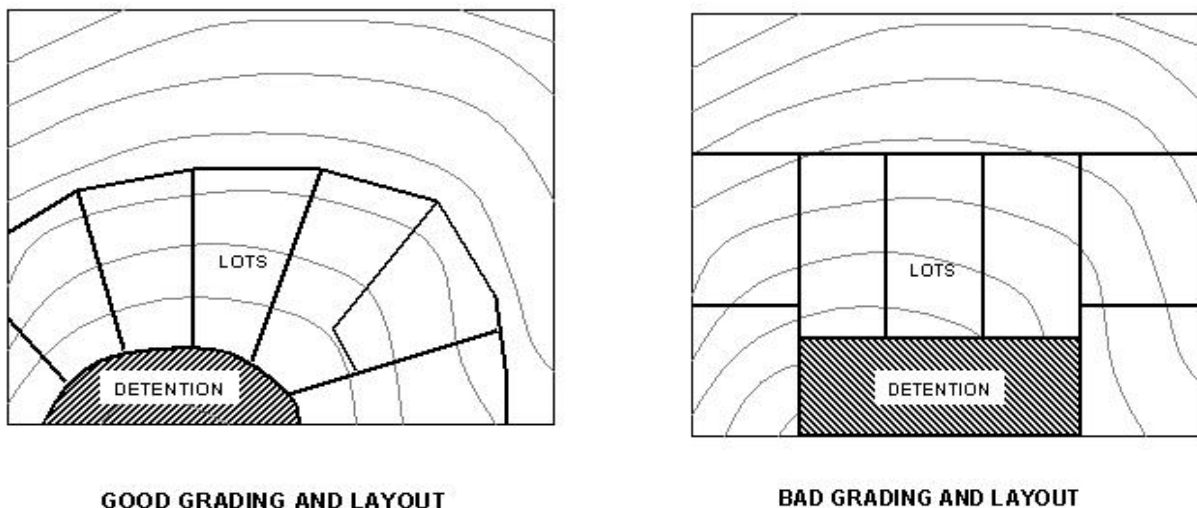
When designing a detention facility, multi-purpose uses, such as active or passive recreation and wildlife habitat, are encouraged in addition to providing the required storage volume. Facilities used for recreation should be designed to inundate no more frequently than every two years.

#### 4.2 Detention Basin Location

Detention basins should be located at the natural low point of the site and must discharge to the natural drainage location to minimize downstream impacts.

#### 4.3 Detention Basin Grading

Detention basin grading shall conform to the natural topography of the site to the maximum extent practical. Developments should be laid out around the existing waterways and proposed detention basin (see Figure DET-3). Layouts conforming to existing topography often reduce construction costs, land disturbance and maintenance costs, and increase aesthetic quality. Existing slopes should be used to the maximum extent practical. If slopes are modified, the maximum allowable slope is 4H:1V. Exceptions to these criteria must be justified through engineering and/or economic studies. Significant modifications to existing topography may require geologic impact studies and geotechnical analysis, particularly where shallow bedrock is believed to be present.



**Figure DET-3**  
**Examples of Good and Bad Location, Grading and Lot Layout for Detention**

#### 4.4 Geometry of Storage Facilities

The geometry of a detention facility depends on specific site conditions such as adjoining land uses, topography, geology, existing natural features, volume requirements, etc. The following criteria apply to the geometry of detention facilities:

- Pond side slopes of 4H:1V are the maximum permissible; slopes between 5H:1V and 10H:1V are encouraged. If slopes steeper than 4H:1V are desired, the engineer must show why 4H:1V slopes are not feasible and provide an explanation regarding how the steeper slopes will be maintained.

- Pond bottom slopes must be a minimum of 1 percent to ensure drainage.
- A concrete low-flow channel shall be provided where the 2-year design flow exceeds 5 cfs. These may not be desirable for water quality basins.
- Hard improvements such as concrete, metal or plastic pipe must be used to control the 2-year design flow. Between the 2- and 10-year design flows, hard armor/grass composites may be considered, provided that velocities are low enough to ensure stability. Above the 10-year water surface, sod, turf reinforcement mat or other composite designs may be used, provided that they are appropriate for design velocities. Sod is acceptable for velocities less than 4 ft/s. Turf reinforcement mat or other composite materials are acceptable for velocities less than 8 ft/s.
- The water quality portion of a facility (if any) should be shaped with a gradual expansion from the inlet and a gradual contraction toward the outlet, thereby minimizing short-circuiting. Storage facility geometry and layout are best developed in concert with a land planner/landscape architect.

#### **4.5 Embankments and Cut Slopes**

If the storage facility is jurisdictional, meaning it is subject to regulation by the MDNR, the embankment shall be designed, constructed, and maintained to meet most current MDNR criteria for jurisdictional structures. The design for an embankment of a storm water detention or retention storage facility should be based upon a site-specific engineering evaluation. The embankment should be designed to prevent catastrophic failure during the 100-year and larger storms. The following criteria apply in many situations (ASCE and WEF 1992):

1. **Side Slopes**—For ease of maintenance, side slopes of the embankment should not be steeper than 3H:1V, with 4H:1V preferred. The embankment's side slopes should be well vegetated, and riprap protection (or the equivalent) may be necessary to protect it from wave action on the upstream face, especially in retention ponds.
2. **Freeboard**—The elevation of the top of the embankment shall be a minimum of 1 foot above the water surface elevation when the emergency spillway is conveying the maximum design or emergency flow. When relevant, all MDNR dam safety criteria must be carefully considered when determining the freeboard capacity of an impoundment.
3. **Settlement**—The design height of the embankment should be increased by roughly 5 percent to account for settlement. All earth fill should be free from unsuitable materials and all organic materials such as grass, turf, brush, roots, and other material subject to decomposition. The fill material in all earth dams and embankments should be compacted to at least 95 percent of the

maximum density obtained from compaction tests performed by the Modified Proctor method in ASTM D698.

4. **Emergency Spillway**—An emergency spillway will usually be needed to convey flows that exceed the primary outlet capacity.

#### **4.6 Linings**

Detention facilities may require an impermeable clay or synthetic liner for a number of reasons. Storm water detention and retention facilities have the potential to raise the groundwater level in the vicinity of the basin. If the basin is close to structures or other facilities that could be damaged by raising the groundwater level, consideration should be given to lining the pond. An impermeable liner may also be warranted in a retention basin where the designer seeks to limit seepage from a permanent pond. Alternatively, there are situations where the designer may seek to encourage seepage of storm water into the ground. In this situation, a layer of permeable material may be warranted.

#### **4.7 Inlets**

Inlets to the facility should incorporate energy dissipation to limit erosion. They should be designed in accordance with drop structure criteria in Chapter 8, Open Channels, or using other approved energy dissipation techniques. In addition, forebays or sediment traps should be incorporated at all inflow points to storage facilities to settle a significant portion of the sediment being delivered by storm water to the facility. Forebays will need regular maintenance to reduce the sediment being transported and deposited on the storage basin's bottom.

#### **4.8 Outlet Works**

Outlet works should be sized and structurally designed to release at the specified flow rates without structural or hydraulic failure. Design guidance for outlet works used for water quality purposes is included in Chapter 10, Water Quality.

#### **4.9 Trash Racks**

Trash racks should be sufficiently sized to not interfere with the hydraulic capacity of the outlet and must be designed in a manner that is protective of public health, safety and welfare. See Chapter 10, Water Quality for minimum trash rack sizes.

#### **4.10 Vegetation**

The type of vegetation specified for a newly constructed storage facility is a function of the frequency and duration of inundation of the area, soil types, whether native or non-native vegetation is desired, and other potential uses (park, open space, etc.) of the area. A planting plan should be developed for new

facilities to meet their intended use and setting in the urban landscape. Generally, trees and shrubs are not recommended on dams or fill embankments.

#### **4.11 Operations and Maintenance**

Maintenance considerations during design include the following (ASCE and WEF 1992):

1. If the detention period is long, and especially if children are apt to play in the vicinity of the impoundment, use of relatively flat side slopes along the banks is advisable. In addition, installation of landscaping that will discourage entry, such as thick, thorny shrubs, is suggested for locations along the periphery, near the outlets, and at steeper embankment sections. If the impoundment is situated at a lower grade than and adjacent to a highway, installation of a guardrail is in order. Providing features to discourage public access to the inlet and outlet areas of the facility should be considered.
2. The facility should be accessible to maintenance equipment for removal of silt and debris and for repair of damages that may occur over time. Easements and/or rights-of-way are required to allow access to the impoundment by the owner or agency responsible for maintenance.
3. Bank slopes, bank protection, and vegetation types are important design considerations for site aesthetics and maintainability.
4. Permanent ponds should have provisions for complete drainage for sediment removal or other maintenance. The frequency of sediment removal will vary among facilities, depending on the original volume set aside for sediment, the rate of accumulation, rate of growth of vegetation, drainage area erosion control measures, and the desired aesthetic appearance of the pond.
5. Adequate dissolved oxygen supply in ponds (to minimize odors and other nuisances) can be maintained by artificial aeration. Use of fertilizer and pesticides adjacent to the permanent pool pond should be carefully controlled.
6. Secondary uses that are incompatible with sediment deposits should not be planned unless a high level of maintenance will be provided.
7. French drains or the equivalent are almost impossible to maintain and should be used with discretion where sediment loads are apt to be high.
8. Underground tanks or conduits designed for detention should be sized and designed to permit pumping. Multiple entrance points should be provided to remove accumulated sediment and trash.

9. Detention facilities should be designed with sufficient depth to allow accumulation of sediment for several years prior to its removal.
10. Permanent pools should be of sufficient depth to discourage excessive aquatic vegetation on the bottom of the basin, unless specifically provided for water quality purposes.
11. Trash racks and/or fences are often used to minimize hazards. These may become eyesores, trap debris, impede flows, hinder maintenance, and, ironically, fail to prevent access to the outlet. On the other hand, desirable conditions can be achieved through careful design and positioning of the structure, as well as through landscaping that will discourage access (e.g., positioning the outlet away from the embankment, etc.). Creative designs, integrated with innovative landscaping, can be safe and can also enhance the appearance of the outlet and pond.
12. To reduce maintenance and avoid operational problems, outlet structures should be designed with no moving parts (i.e., use only pipes, orifices, and weirs). Manually and/or electrically operated gates should be avoided. To reduce maintenance, outlets should be designed with openings as large as possible, be compatible with the depth-outflow relationships desired, and be designed with water quality, safety, and aesthetic objectives in mind. Outlets should be robustly designed to lessen the chances of damage from debris or vandalism. The use of thin steel plates as sharp-crested weirs should be avoided because of potential accidents, especially with children. Trash racks must protect all outlets, especially ones made of a thin plate.
13. Clean out all forebays and sediment traps on a regular basis or when routine inspection shows them to be  $\frac{3}{4}$  full.

See Chapter 10, Water Quality, for additional recommendations regarding operation and maintenance of water quality related facilities, some of which also apply to detention facilities designed to meet other objectives.

#### **4.12 Access**

All-weather, stable access to the bottom, inflow, forebay, and outlet works areas shall be provided for maintenance vehicles. Maximum grades should be 10 percent, and a solid driving surface of gravel, rock, concrete, or gravel-stabilized turf should be provided.

#### **4.13 Geotechnical Considerations**

The designer must account for the geotechnical conditions of the site. These considerations may include issues related to embankment stability, geologic hazards, seepage, and other site-specific issues. It may be necessary to confer with a qualified geotechnical engineer during both design and construction, especially for larger detention and retention storage facilities.

#### **4.14 Environmental Permitting and Other Considerations**

The designer must account for environmental considerations surrounding the facility and the site during its selection, design and construction. These can include regulatory issues such as a) whether the facility will be located in a jurisdictional wetland, b) whether the facility is to be located on a waterway regulated by the USACE as a “Water of the U.S.,” and c) whether there are threatened and endangered species or habitat in the area. See Chapters 2 and 3 for more information on regulatory and permitting requirements.

There are also non-regulatory environmental issues that should be taken into account. Detention facilities can become breeding grounds for mosquitoes unless they are properly designed, constructed and maintained. Area residents may object to facilities that impact riparian habitat or wetlands. Considerations of this kind must be carefully taken into account, and early discussions with relevant federal, state and local regulators are recommended.

### **5.0 EXAMPLES**

The examples provided in this section are calculated step-by-step to provide an understanding of the underlying engineering principles. Worksheets in the SF-Detention Spreadsheet can also be used to complete these calculations, but are not used in these examples.

#### **5.1 Rational Formula Based Modified FAA Procedure**

Use the Rational Formula Based Modified FAA Procedure to determine the required detention volume for the 100-year storm event for a 40-acre watershed, based on single-family land use. The watershed has a 100-year runoff coefficient of 0.56 and a time of concentration of 25 minutes. The post-development 100-year, undetained peak flow rate from the watershed is 157 cfs. Runoff calculations for this example are included in the Rational Method calculation example Chapter 5, Calculation of Runoff. The pre-project 100-year peak flow rate for the 100-year event is 90 cfs.

Given the information above, the following variables are known:

$$A = 40 \text{ acres}$$

$$C = 0.56$$

$$Q_{po} = 90 \text{ cfs}$$

$$t_c = 25 \text{ minutes}$$

Following the methodology outlined in Section 3.2.2, Table DET-4 can be created to determine the required detention volume.

**Table DET-4**  
**Rational Formula Based Modified FAA Procedure Example**

Rainfall Duration (min)	Rainfall Intensity (in/hr)	Inflow Volume (ft <sup>3</sup> )	Adjustment Factor	Average Outflow (cfs)	Outflow Volume (ft <sup>3</sup> )	Storage Volume (ft <sup>3</sup> )
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	-----	-----	-----	-----	-----	-----
5	11.76	79027	1	90.0	27000	52027
10	10.32	138701	1	90.0	54000	84701
15	8.84	178214	1	90.0	81000	97214
20	7.91	212710	1	90.0	108000	104710
25	6.99	234752	1	90.0	135000	99752
30	6.06	244339	0.92	82.5	148500	95839
35	5.69	267658	0.86	77.1	162000	105658
40	5.32	286003	0.81	73.1	175500	110503
45	4.95	299376	0.78	70.0	189000	110376
50	4.58	307776	0.75	67.5	202500	105276
55	4.21	311203	0.73	65.5	216000	95203
60	3.84	309658	0.71	63.8	229500	80158
65	3.72	324761	0.69	62.3	243000	81761
70	3.60	338218	0.68	61.1	256500	81718
75	3.47	350028	0.67	60.0	270000	80028
80	3.35	360192	0.66	59.1	283500	76692
85	3.23	368710	0.65	58.2	297000	71710
90	3.11	375581	0.64	57.5	310500	65081
95	2.98	380806	0.63	56.8	324000	56806
100	2.86	384384	0.63	56.3	337500	46884
105	2.74	386316	0.62	55.7	351000	35316
110	2.62	386602	0.61	55.2	364500	22102
115	2.49	385241	0.61	54.8	378000	7241
120	2.37	382234	0.60	54.4	391500	-9266

Column (1) Storm duration (*t*) in 5-minute increments (typical)

Column (2) Intensity (*i*) for storm duration (*t*) from intensity-duration-frequency table in Chapter 5,

Calculation of Runoff. Note: some values are from linear interpolation of tabular data.

Column (3) =  $C \cdot \text{Col (2)} \cdot A \cdot 60 \cdot \text{Col (1)} = 0.56 \cdot \text{Col (2)} \cdot 40 \cdot 60 \cdot \text{Col (1)}$  [Equation DET-1]

Column (4) =  $0.5 \cdot (1 + t_c / \text{Col (1)}) = 0.5 \cdot (1 + 25 / \text{Col (1)})$  [Equation DET-2]

Column (5) =  $\text{Col (4)} \cdot Q_{po} = \text{Col (4)} \cdot 90$  [Equation DET-3]

Column (6) =  $\text{Col (5)} \cdot 60 \cdot \text{Col (1)}$  [Equation DET-4]

Column (7) =  $\text{Col (3)} - \text{Col (6)}$  [Equation DET-5]

The required detention volume is determined from the maximum storage volume (column 7) in Table DET-4. For this example, the required detention volume is 110,503 ft<sup>3</sup> or 2.5 acre-feet. Because this volume exceeds the 20,000-ft<sup>3</sup> threshold for applicability of the FAA method for final detention sizing, this

should be treated as an initial estimate, and a hydrograph-based method should be used to determine detention storage requirements.

## 5.2 Simplified Detention Storage Calculation

Given an inflow hydrograph for a 20-acre commercial site (calculated according to guidelines in Chapter 5, Calculation of Runoff) and a maximum allowable release rate of 30 cfs, determine the preliminary detention volume required. The tabular format for use with the inflow hydrograph method is shown in Table DET-5 below. The time and flow ordinates of the inflow hydrograph are entered in columns 1 and 2 of Table DET-5. Based on the inflow hydrograph, the allowable release rate of 30 cfs is matched on the falling limb at a time between 102 and 108 minutes, so 108 minutes is used as an estimate for  $T_p$ .

**Table DET-5**  
**Simplified Detention Volume Calculation Example**

Time (min)	Inflow Hydrograph (cfs)	Outflow Rising Hydrograph (cfs)	Incremental Storage Volume (acre-feet)	Cumulative Storage Volume (acre-feet)
(1)	(2)	(3)	(4)	(5)
0	0	0	0.00	0.00
6	0	2	0.00	0.00
12	5	3	0.02	0.02
18	41	5	0.30	0.31
24	97	7	0.75	1.06
30	128	8	0.99	2.05
36	130	10	0.99	3.05
42	122	12	0.91	3.95
48	107	13	0.78	4.73
54	91	15	0.63	5.36
60	77	17	0.50	5.86
66	66	18	0.40	6.26
72	56	20	0.30	6.56
78	45	22	0.19	6.75
84	37	23	0.12	6.87
90	33	25	0.07	6.94
96	31	27	0.04	6.98
102	30	28	0.02	7.00
108	30	30	0.00	7.00
114	28			

Columns (1) & (2) Input from SCS Unit Hydrograph analysis with Huff distribution

Column (3) =  $(T/T_p) \cdot Q_{po} = (\text{Col}(1)/108) \cdot 30$  [Equation DET-6]

Column (4) =  $((\text{Col}(2) - \text{Col}(3)) \cdot 60 \cdot 6) / 43560$ . (includes unit conversion) Note: if  $\text{Col}(2) - \text{Col}(3) < 0$ , then  $\text{Col}(4) = 0$ .

Column (5) =  $(\text{Col}(5) \text{ Row } (i-1)) + (\text{Col}(4) \text{ Row } (i))$

### 5.3 Modified Puls Reservoir Routing Example

Given the inflow hydrograph from the example in Section 5.2 for a 20-acre commercial site, a detention basin with the stage-storage relationship in Table DET-6 is proposed.

**Table DET-6**  
**Stage-Storage Relationship for Detention Facility**

Stage (elevation [ft] above mean sea level)	Storage (acre feet)
1320	0
1321	0.5
1322	1.5
1323	4.0
1324	7.0
1325	10.0

The stage-outflow relationship for the detention facility outlet structure (determined from hydraulic analysis) is summarized in Table DET-7.

**Table DET-7**  
**Stage-Outflow Relationship for Detention Facility**

Stage (elevation [ft] above mean sea level)	Outflow (cfs)
1320	0
1321	5
1322	10
1323	20
1324	30
1325	40

The following steps are used to determine the outflow hydrograph for this proposed facility following the guidance in Section 3.3.3.1:

1. Determine the inflow hydrograph: The inflow hydrograph should be developed following guidance in Chapter 5, Calculation of Runoff.
2. Select a routing interval ( $\Delta t$ ): A rule of thumb for selecting the routing interval is to divide the rising limb of the hydrograph into 10 increments. Since it takes about 40 minutes for the hydrograph to peak, use a routing interval of 4 minutes.
3. Establish the storage-outflow relationship as shown in Table DET-8:

**Table DET-8**  
**Storage-outflow Relationship for Detention Facility**

Stage (elevation [ft] above mean sea level)	Outflow (O) (cfs)	Storage (S) (acre-feet)	$2S/\Delta t$ (cfs)	$2S/\Delta t + O$ (cfs)
(1)	(2)	(3)	(4)	(5)
1320	0	0.0	0	0
1321	5	0.5	182	187
1322	10	1.5	545	555
1323	20	4.0	1452	1472
1324	30	7.0	2541	2571
1325	40	10.0	3630	3670

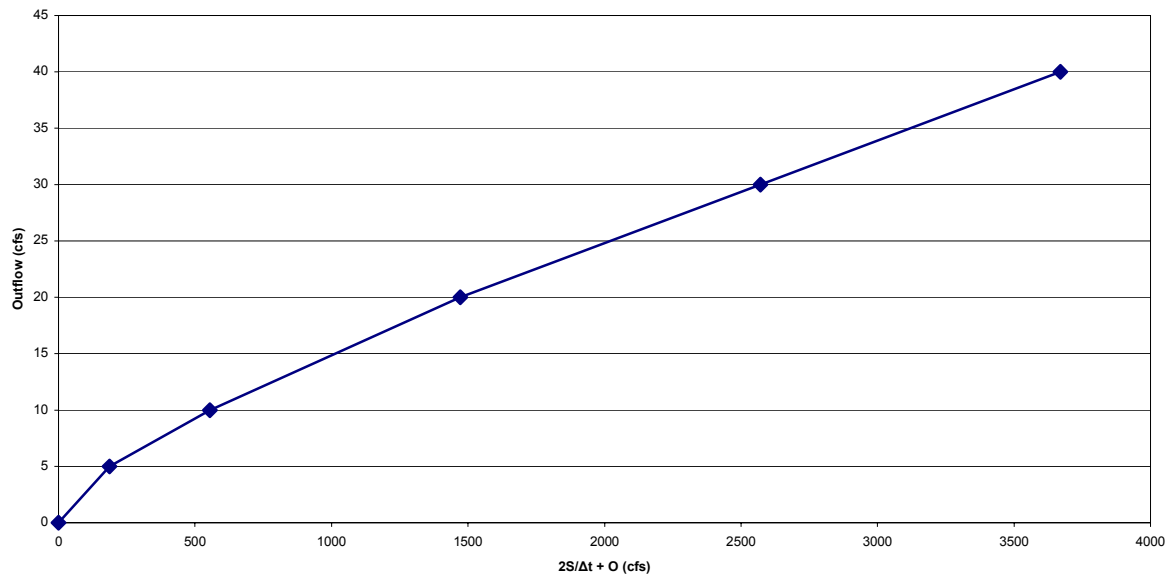
Columns (1) and (3) from Table DET-6

Columns (1) and (2) from Table DET-7

Column (4) =  $2S/\Delta t$  \* (unit conversion) =  $2 * \text{Col (3)} / (4 \text{ min} * 60 \text{ sec/min}) * (43560 \text{ ft}^2/\text{acre})$

Column (5) = Col (4) + Col (2)

4. Plot the  $(2S/\Delta t) + O$  versus  $O$  relationship: Plot values from Table DET-8. This relationship is shown in Figure DET-4.



**Figure DET-4**  
 **$2S/\Delta t + O$  versus  $O$  for Reservoir Routing Example**

5. Perform the Modified Puls routing using a table:

An example of the Modified Puls routing method is shown in Table DET-9. Table heading descriptions are provided following the table.

**Table DET-9**  
**Modified Puls Routing Table**

Time (min)	Inflow ( $I_i$ ) (cfs)	Inflow ( $I_j$ ) (cfs)	$2S/\Delta t - O$ (cfs)	$2S/\Delta t + O$ (cfs)	Outflow ( $O$ ) (cfs)
(1)	(2)	(3)	(4)	(5)	(6)
0	0.00	0.01	0	--	0
4	0.01	0.59	0.01	0.01	0.0006
8	0.59	5.40	0.59	0.62	0.02
12	5.40	25.61	6.23	6.58	0.18
16	25.61	60.13	35.24	37.23	1.00
20	60.13	97.40	114.48	120.97	3.24
24	97.40	121.10	259.69	272.01	6.16
28	121.10	130.28	460.26	478.19	8.96
32	130.28	130.03	688.22	711.64	11.71
36	130.03	124.85	919.94	948.53	14.29
40	124.85	117.18	1141.29	1174.81	16.76
44	117.18	107.44	1345.25	1383.32	19.03
48	107.44	96.71	1528.09	1569.87	20.89
52	96.71	86.37	1687.50	1732.24	22.37
56	86.37	77.29	1823.33	1870.58	23.63
60	77.29	69.90	1937.62	1986.99	24.69
64	69.90	63.07	2033.65	2084.81	25.58
68	63.07	56.02	2113.98	2166.62	26.32
72	56.02	48.75	2179.22	2233.07	26.93
76	48.75	42.31	2229.21	2283.99	27.39
80	42.31	37.42	2264.82	2320.26	27.72
84	37.42	34.42	2288.67	2344.55	27.94
88	34.42	32.54	2304.35	2360.52	28.08
92	32.54	31.38	2314.95	2371.31	28.18
96	31.38	30.72	2322.37	2378.87	28.25
100	30.72	30.30	2327.86	2384.46	28.30
104	30.30	29.96	2332.19	2388.88	28.34
108	29.96	29.24	2335.70	2392.46	28.38
112	29.24	26.98	2338.11	2394.90	28.40
116	26.98	24.08	2337.55	2394.33	28.39
120	24.08	21.58	2331.93	2388.61	28.34
124	21.58	19.40	2321.11	2377.59	28.24
128	19.40	16.20	2305.90	2362.09	28.10
132	16.20	11.82	2285.67	2341.49	27.91
136	11.82	7.66	2258.37	2313.69	27.66
140	7.66	4.56	2223.20	2277.86	27.33
144	4.56	2.83	...	...	...

Columns 1-3 are known inputs into the table, and the rest of the columns are unknown (blank) when the routing process begins. The objective is to complete the last column, which represents the outflow hydrograph. Inputs and calculations for each column include:

- **Column 1** (time) and **Column 2** (inflow) provide the design inflow hydrograph (obtained using methods described in Chapter 5, Calculation of Runoff).
- **Column 3** is the value from column 2 moved earlier in time (up the table) one time increment.
- **Column 4:** To initiate the routing process with little or no inflow, assume the initial value is 0. The next value of  $2S/\Delta t - O_j$  confirms this assumption. Subsequent values of  $(2S/\Delta t) - O$  are calculated by doubling the outflow values in column 6 and subtracting them from  $(2S/\Delta t) + O$ .
- **Column 5:** The values in column 5 are calculated by applying the continuity equation (storage relationship) in Equation DET-19:

$$(I_i + I_j) + \left[ \frac{2S_i}{\Delta t} - O_i \right] = \left[ \frac{2S_j}{\Delta t} + O_j \right]$$

For the first time increment (4 minutes), this is:  $(0 + 0.01) + [0] = [0.01]$ .

- **Column 6:** The first value of outflow is assumed to be equal to inflow. Subsequent values are obtained from the  $(2S/\Delta t) + O$  versus  $O$  relationship in Figure DET-4 and Table DET-9. Linear interpolation can be used to determine  $O$  values for a given  $(2S/\Delta t) + O$  using Table DET-9 for values that cannot be easily read from Figure DET-4 (for the first row of Column 6, see Step 2 above).

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